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Shamsher Prakash
Missouri University of Science and Technology, prakash@mst.edu

M. K. Gupta
University of Roorkee, Roorkee, India

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Liquefaction of a Soil Deposit During an Earthquake

M. K. Gupta
Professor, Earthquake Engineering Department, University of Roorkee, Roorkee, India

Shamsher Prakash
Professor in Civil Engineering, University of Missouri-Rolla (USA) and Director, Central Building Research Institute, Roorkee, India, Formerly, Professor and Head, Civil Engineering Department, University of Roorkee, Roorkee, India

SYNOPSIS Large number of cases have been reported where liquefaction has occurred during the earthquake, but only in a few cases, soil report in the zones of liquefaction are available. A wide spread damage because of liquefaction of soil deposit was observed during the Niigata Earthquake of 1964 in Japan. A very systematic study on soil exploration in the zone of liquefaction was carried out and is well reported. Two sites were selected for the analysis from Niigata in the same area (i) where heavy damage occurred (ii) where no damage occurred. A case study was made using two different methods having different philosophy of analysis for prediction of possibility of liquefaction during an earthquake. The results of different methods are in good agreement with the observed behaviour where liquefaction was observed during the earthquake. But a wide controversy is observed between the two methods where liquefaction did not occur. The paper presents the case study.

INTRODUCTION

Liquefaction of saturated sand has often been the main cause of catastrophic damage to structures resulting in loss of life and property. This has been amply demonstrated during many earthquakes especially the Niigata Earthquake of 1964. On examination of damage during the past earthquakes, it can be summarized that liquefaction of foundation soil may cause (i) Small or large land slides, (ii) Displacements, large settlements, overturning and sinking of buildings and other heavy civil engineering structures like bridge piers and abutments, harbour facilities and water towers, (iii) Cracking and slumping of embankments, (iv) Floating up of buried structures like wooden piles, septic and storage tanks, sewage conduits, manholes and water mains, (v) Filling up of wells and ditches with sand and lifting of canal beds, (vi) Sinking of heavy equipment placed on such soils like automobiles and machinery, (vii) Development of fissures and sand boils through which sand and water may be brought up from depths,(viii) General subsidence and inundating of area from the ejected water out of ground.

One of the important aspect of liquefaction during the earthquake is the progressive development of liquefaction and spreading of liquefaction zone. Liquefaction of a soil layer in first few cycles of ground motion would reduce the overburden pressure on lower layers. This may cause favourable condition for liquefaction to develop in the under lying layer in the next few cycles, there by further reducing the overburden pressure on deeper layers. In this way the liquefaction may travel to sufficient deep layers during the earthquake.

Basically two types of approachs are available to determine the possibility of liquefaction of a site during an expected earthquake. One considers the progressive nature of development of liquefaction step by step and the earthquake is considered to consist of suitable different number of cycles of different intensities of accelerations keeping the sequence of vibrations the same as in the accelerogram of the expected earthquake (Gupta 1977). The second is based on the philosophy of converting the entire history of earthquake into an equivalent number of cycles of an average uniform intensity of ground motion (Seed and Idriss 1971).

A case study was made for the Niigata site where wide spread liquefaction was observed during the Niigata Earthquake of June 16, 1964 and where well planned soil exploration report is also available. The study was planned to demonstrate the above two types of approaches. Two locations were selected. At one location the sand had liquefied while at the other location the sand had not liquefied. This study shows that the first approach which considers progressive nature has predicted the field behaviour for both the places where liquefaction had occurred and where it had not occurred while the second approach could predict the behaviour only where the sand had liquefied. The paper presents the details of the case study.

LIQUEFCTION ANALYSIS

Approach One

This approach involves an estimate of loss of effective overburden pressure step by step in a progressive way, on account of increase in pore pressure during the earthquake. The following physical phenomenon constitute the basis of analysis of liquefaction.

a) If a pressure p_0 is applied on top of a column of water, the water pressure is increased equally by p_o throughout the column height.

Therefore an increase in pore water pressure during vibrations at a depth in a saturated soil mass causes an equal increase in pore water pressure throughout the deposit below this depth. This phenomenon has been observed in the laboratory on horizontal vibration table tests by Florin and Ivanov (1961).

b) If some pore pressures are developed in first few cycles of ground motion at a depth, these will be transmitted equally in all the deeper layers and the effective overburden pressure in these deeper layers is reduced. In the next few cycles and under reduced effective overburden pressure further pore water pressure may develop in deeper layer resulting in still further reduction in effective overburden pressure on still deeper layers. In this process, if the pore water pressure becomes equal to the initial effective over burden pressure, the liquefaction may occure starting at some depth and moving to deeper layers.

c) The pore water pressure developed during the earthquake dissipate by upward movement of water, thus setting up a water flow. During this process a hydraulic gradient is set up in the deposit and may develop quick conditions in the upper layers which may not have liquefied earlier. In many cases for all practical purposes they have been seen to liquefy during this vertical flow of water forming vertical pipes in the deposit and bringing out the sand and water mixture.

Based on this physical concept a method of analysis is developed which considers the progressive nature of liquefaction development (Gupta 1977). The information required and the method of analysis is given below.

Information Required for the Analysis

The following information is required for carrying out the analysis.

1. Depths of alluvial deposits and their relative densisites and position of water table in field - Depth of deposits and position of water table can be obtained by soil exploration at the site and relative densities by correlating the N value (S.P.T. test data) with relative density.

2. Effective overburden pressure with depth - This can be computed by assuming a reasonable value of density or from engineering properties obtained on sand samples in laboratory for the purpose of this analysis.

3. Accelerogram of the anticipated earthquake - Estimation of an appropriate accelerogram for any site is a difficult problem. However a suitable accelerogram for a site can be asigned by considering the upward propagation of shear waves from an under lying firm base or by directly modifying an accelerogram of a past earthquake if possible.

4. Laboratory data - After the relative densities and effective overburden pressure with depth and accelerogram for the site are established, the laboratory tests are required to be performed on the sand sample obtained from the site on a vibration table (Gupta and Prakash 1977). The tests are carried out at predominent frequency of the accelerogram at different relative densities estimated for field and different dead weight surcharges. Plots of excess pore water pressure versus effective overburden pressure for corresponding field acceleration and relative densities (D_{ri}) are obtained. These plots will serve as laboratory data for making the analysis. One such plot is shown in Figure 1.

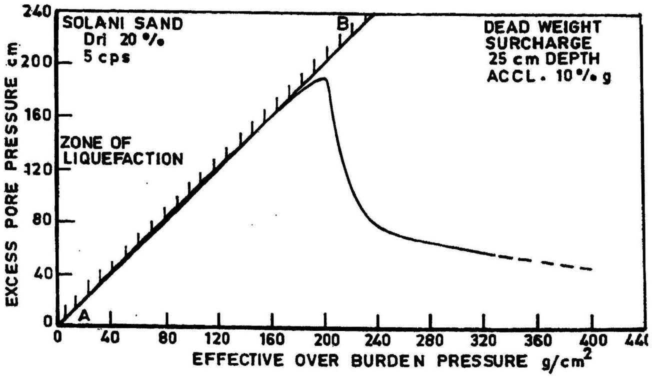

Fig. 1. Pore Pressure Vs Effective Overburden

Liquefaction Analysis

After obtaining the necessary information the analysis is performed step by step as shown in Figure 2. The steps involved are described below.

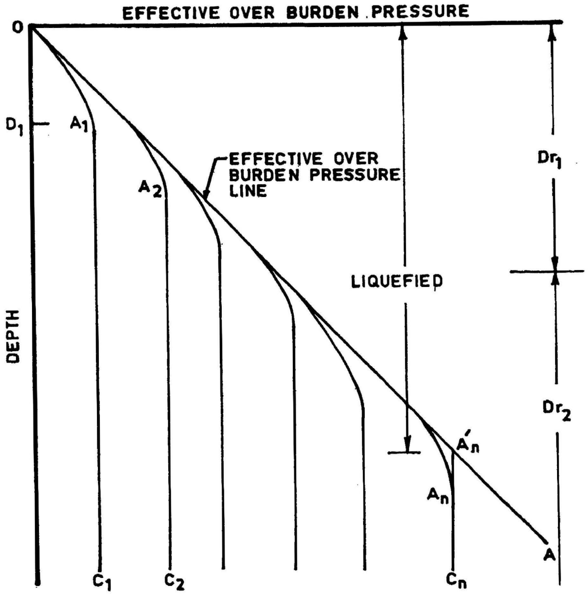

Fig. 2. Pore Water Pressure During Earthquake

1. Mark the thickness of different strata of different relative densities.
2. Plot the effective overburden pressure versus depth as shown by line CA.
3. Consider the accelerogram to consist of suitable different number of cycles of different intensities of accelerations as below keeping the sequence of vibrations the same as in the accelerogram

Acceleration Intencity	a_1	a_2	a_3	$\cdots a_n$
Number of cycles	n_1	n_2	n_3	$\cdots n_n$

4. For first set of n_1 cycles of acceleration (a_1) the pore pressure with depth is plotted with the help of corresponding laboratory data till maximum pore pressure at a particular depth D_1 is obtained. The pore pressure below this depth D_1 equals this maximum value as explained earlier. Thus pore pressure line in first n_1 cycles is given by OA_1C_1. Hence the horizontal ordinate between OA and CA_1C_1 on any particular depth represents the effective overburden pressure immediately after n_1 cycles of attendant motion.
5. Repeat step 4 for second set of number of cycles n_2 of intensity a_2 and obtained further loss in effective overburden given by CA_2C_2.
6. Repeat the process step by step for all the sets of cycles finally obtaining a loss in effective overburden given by CA_nC_n.
7. Extend A_nC_n to meet OA at A_n'. Depth to A_n' has been considered to have liquefied in this analysis.

Prediction of liquefaction possibility in field is possible in a more rational manner in the above analysis. This method considers the progressive failure of ground on account of liquefaction as expected during ground shaking. The case of the Niigata earthquake of June 16, 1964 was studied for the two locations as given below.

LIQUEFACTION ANALYSIS OF NIIGATA SITE

Two locations were selected for analysis from Niigata. In location I there was heavy damage on account of liquefaction during the earthquake which has been estimated to be of 7.5 magnitude at an epicentral distance of about 50 km. Location II also lies in the same area but there was a 2.7 m high dry fill. No damage occurred at this place.

A detailed laboratory tests on horizontal vibra ation table were carried out on a locally available Solani sand (Gupta 1977). The range of grain size of deposits at Niigata are shown in Figure 3. In the same figure the grain size distribution of Solani sand is also compared. Since this lies fairly close to the Niigata sand, laboratory data obtained on this sand may be assumed to be reasonably applicable for the liquefaction analysis of Niigata area.

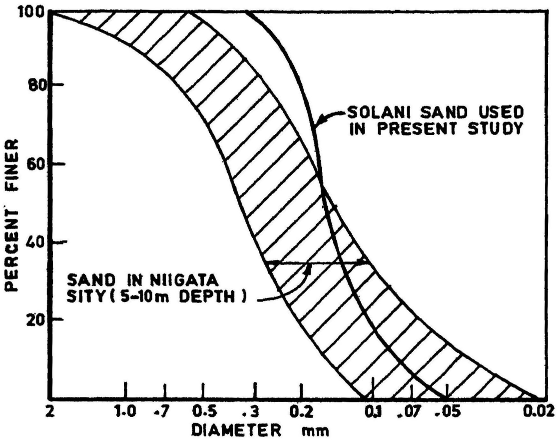

Fig. 3. Grain Size Distribution

For making the liquefaction analysis of a site, the suitable accelerogram of the expected earthquake is required. The ground motion developed near the surface during an earthquake may be attributed primarily to the upward propagation of shear waves from an underlying firm base. If the ground surface and the underlying rock surface are horizontal, the time wise response of ground may be obtained by considering it as one dimensional shear beam, when the base rock is under seismic excitation (Idriss and Seed 1968).

The thickness of alluvial deposits in Niigata area has been reported to be about 80-160m above the firm base (Kawasumi 1968). To determine a suitable accelerogram for the Niigata site a time wise shear beam response of the deposit was carried out considering the firm base at depths of about 80m. The ground response was determined when the firm base below was subjected to a modified Koyna earthquake duly accounted for Magnitude of Earthquake and the duration (Gupta 1977). The ground response thus determined constitute the artificial earthquake for the liquefaction analysis and is shown in Figure 4. The duration of this earthquake is about 25 seconds and it contains about 70 cycles of motion. Approximately for about first 30 cycles, acceleration levels are about 10%g or more. For the next 20 cycles the accelerations are of the order of 5%g and for next 20 cycles it varies from about 10% to 5%g. Laboratory tests on Solani sand (similar to Niigata area) show that it liquefies completely at acceleration level of 10%g or more and maximum pore water pressure developed is same under this condition. Therefore, if all

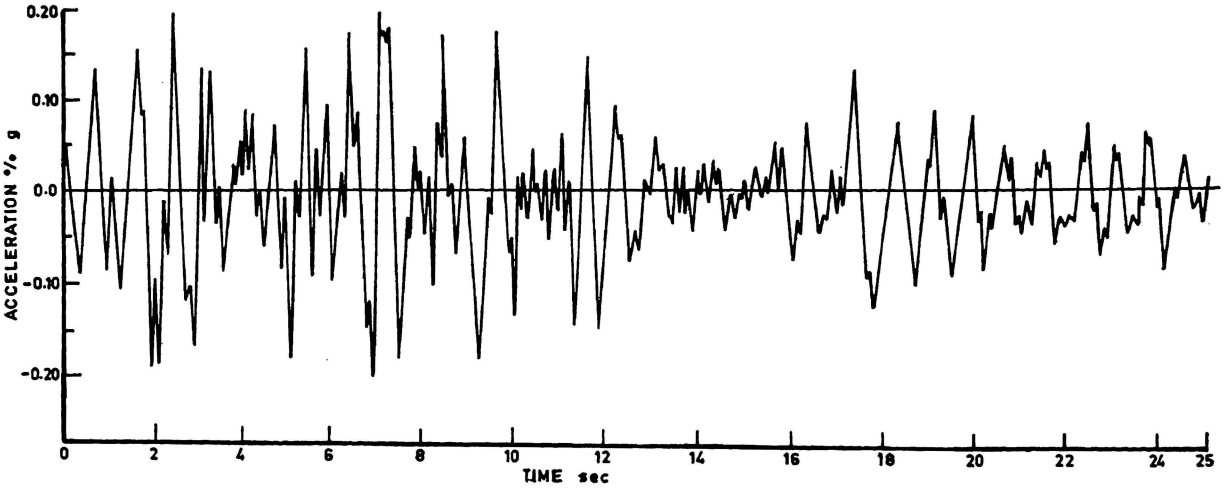

Fig. 4. Accelerogram

the peaks of 10%g or more are considered equivalent to 10%g intensity, no error in estimation of pore water pressure would be committed. It is also shown that maximum pore pressure at any acceleration level may develop in about 10 cycles (Gupta 77). Therefore without much error involved, the above accelerogram may be considered to consist of following sequence of combination of acceleration and number of cycles.

Acceleration %g	10	10	10	5	5	10	5
Number of Cycles	10	10	10	10	10	10	10

Analysis for Location I

The liquefaction analysis is shown in figure 5 and illustrated in table I. The effective overburden is plotted with depth in this figure. The loss in overburden pressure is estimated for the above combinations of accelerations and number of cycles step by step as explained earlier. After first 10 cycles of 10% g i.e. after about 3.6 sec., a maximum loss of 190 g/cm^2 in overburden occurs in a layer with 22% relative density, at a depth of 2.27 metres below the ground surface and the overburden pressure on deeper layers is lost uniformly by this amount. For the next 10 cycles a maximum loss of 165 g/cm^2 in overburden occurs in layer with 36% relative density at a depth of 4.25m. Thus proceeding step by step for all the sets of cycles a total loss of 1040 g/cm^2 in overburden pressure takes place and an effective overburden of 40 g/cm^2 i.e. only 3.7% of initial overburden of 1080 g/cm^2 at 12m depth is left. This analysis indicate that soil deposit would liquefy to depths of about 11.5m during the earthquake. The reported depth of liquefaction at this site is 10m to 15m (Kawasumi 1968). This shows that the method proposed above predicts the liquefaction of a soil deposit in field resonably well.

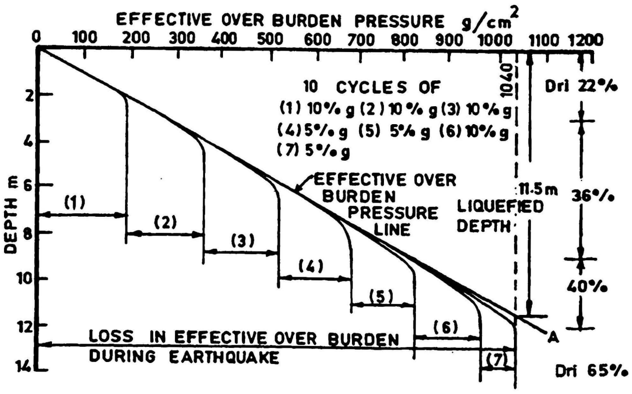

Fig. 5. Liquefaction Analysis of Niigata Site

TABLE I. LIQUEFACTION ANALYSIS OF NIIGATA SITE

Sl. No.	Accelera-tion %g	No. of cycles	Depth m	Effective over-burden g/cm^2	Excess pore pressure g/cm^2	Residual over burden g/cm^2
1	2	3	4	5	6	7
1	10	10	1	90	92	-
			2	180	180	-
			4	360	190	170
			6	540	190	350
			8	720	190	530
			10	900	190	710
			12	1080	190	890
(Maximum pore pressure at depth 2.27m)						
2	10	10	1	-	-	-
			2	-	-	-
			4	170	148	22
			6	350	165	185
			8	530	165	365
			10	710	165	545
			12	890	165	725
(Maximum pore pressure at depth 4.25m)						
3	10	10	1	-	-	-
			2	-	-	-
			4	22	18	4
			6	185	160	25
			8	365	165	200
			10	545	165	380
			12	725	165	560
(Maximum pore pressure at depth 6.15m)						
4	5	10	1	-	-	-
			2	-	-	-
			4	4	3.5	-
			6	25	18	7
			8	200	160	40
			10	380	160	220
			12	560	160	400
(Maximum pore pressure at depth 7.95m)						
5	5	10	1	-	-	-
			2	-	-	-
			4	-	-	-
			6	7	4	3
			8	40	24	16
			10	220	135	85
			12	400	135	265
(Maximum pore pressure at depth 9.75m)						
6	10	10	1	-	-	-
			2	-	-	-
			4	-	-	-
			6	3	2	1
			8	16	10	6
			10	85	57	28
			12	265	145	120
(Maximum pore pressure at depth 11.35m)						
7	5	10	1	-	-	-
			2	-	-	-
			4	-	-	-
			6	-	-	-
			8	6	4	2
			10	28	16	12
			12	120	80	40
(Maximum pore pressure at depth 12 m)						

Analysis for Location II

liquefaction analysis for this location is shown in Figure 6 and table II illustrates it. The analysis shows that after the earthquake residual effective overburden pressure at the level of the water table is about 108 g/cm^2 i.e. about 23% of initial effective overburden of 480 g/cm^2 and a pore pressure of about 372 g/cm^2 has built up during the earthquake. When this will try to dissipate it can cause only a small temporary rise in water table and no damage is expected because of dry soil. This result is in conformity with the observed field behaviour.

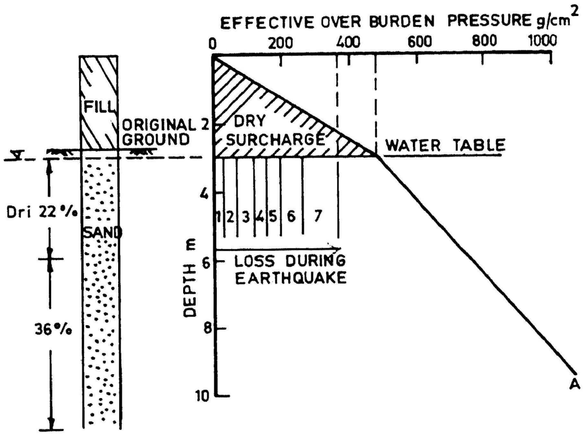

Fig. 6. Liquefaction Analysis of Fill Area

TABLE II. LIQUEFACTION ANALYSIS OF FILL AREA

Sl. No.	Accelera-tion %g	No. of cycles	Depth m	Effective over-burden pressure g/cm^2	Excess pore P-ressure g/cm^2	Residual over burden pressure g/cm^2
1	2	3	4	5	6	7
1	10	10	3	480	35	445
2	10	10	3	445	42	403
3	10	10	3	403	48	355
4	5	10	3	355	35	320
5	5	10	3	320	40	280
6	10	10	3	280	67	213
7	5	10	3	213	105	108

APPROACH TWO

Location I

The analysis was carried out for this location with the method proposed by Seed and Idriss (1971) and is presented in Figure 7. This method indicate that liquefaction may occur upto a depth of about 16 m. These results are in good agreement with the observed behaviour.

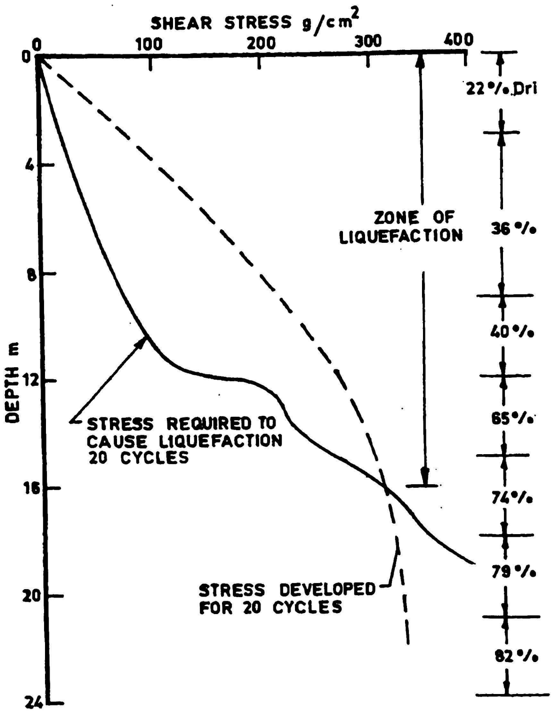

Fig. 7. Liquefaction Analysis of Niigata Site

Location II

The analysis for this site was also carried out by Seed and Idriss (1971) approach and is presented in Figure 8, where it predicts liquefaction to a depth of about 17 m. While no traces of liquefaction were observed at this site.

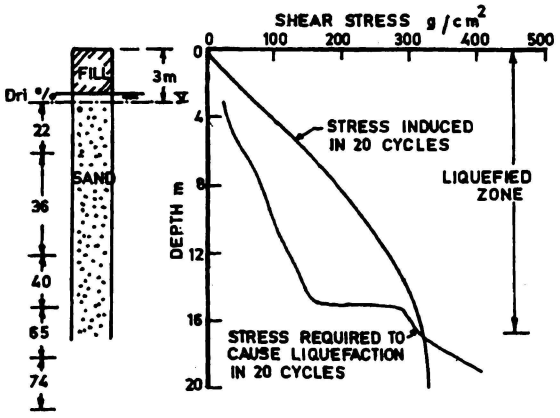

Fig. 8. Liquefaction Analysis of Fill Area

The vibration table test (Approach one) depicts the field behaviour to a reasonably good extent In these tests the sample is prepared and consolidated under anisotropic condition similar to field. Also the deformations occur under plane strain conditions. The method using the vibration table test data has predicted field behaviour for both places where liquefaction had occurred and where it had not occurred. This reflects sufficient confidence in this method of analysis and in the attendent vibration table tests which are much simpler to perform and monitor a compared to the triaxial tests. It is importan to note that the proposed method of analysis determines the possibility of liquefaction by progressive failure as expected in field.

CONCLUSIONS

1. Vibration table tests are considered to represent the field condition with sufficient degree of confidence. These are much simpler to perform and monitor.

2. A method has been developed for making the analysis which considers progressive nature of liquefaction in a more rational manner and has been shown to be more realistic and reliable.

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